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foundation of the West Bridge

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Great Belt - Foundation of the West Bridge

by

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Synopsis: This paper describes aspects of the soil investigations and geotechnical evaluations for the foundation design of the 6.6 km long Great Belt West Bridge. The gravity foundations rest predominantly on glacial tills and prequaternary limestone. Special investigations for assessment of the soil properties for ship impact and ice loading are described briefly, and first experiences from settlement monitoring of the structure during erection are presented.

INTRODUCTION

The Great Belt is part of the inland sea area and approximately 18 km wide. It divides Denmark's population in two halves. In 1986 the Government established a political agreement for construction of a fixed link for both rail and road traffic across the Belt.

As the Belt is separated into two channels by the tiny island of Sprogø, it was decided to establish the link in three major structures: a bored railway tunnel and a high level road bridge across the eastern channel, and a low level bridge for combined traffic across the western channel, see Fig. 1.

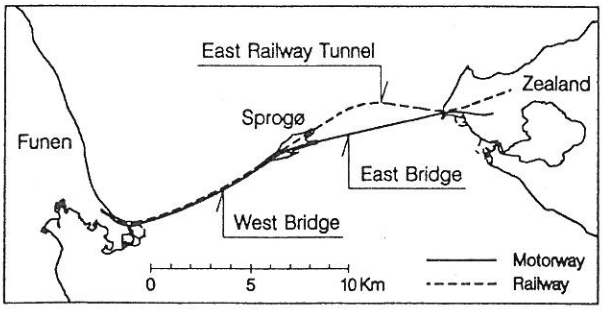

Fig. 1. The Great Belt Link

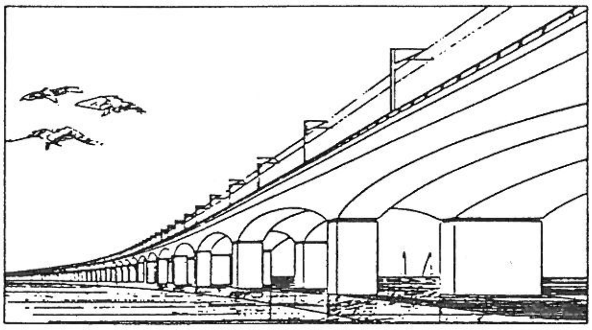

Fig. 2. The West Bridge

A limited company, Great Belt A.S., was founded by the Danish state with the objective of building and operating the link.

THE WEST BRIDGE

The overall concept for the 6.6 km long twin girder concrete bridge, see Fig. 2, is based on prefabrication of all main

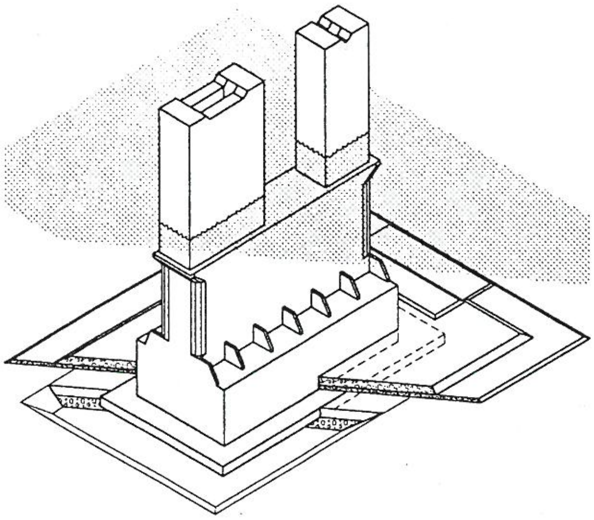

Fig. 3. Substructure

elements. The bridge consists of two parallel box girders supported on separate pier shafts which share a common substructure designed as a gravity founded caisson.

The bridge elements are cast in a reclaimed harbour area. Transportation and installation of the elements are performed by a large catamaran crane vessel.

The girders weighing up to 5700 tons are placed on temporary bearings. After the mid span joints are cast and continuity is established, the girders are adjusted and the permanent bearings connected. Regular spans are 110 m, and 12 expansion joint spans are 82 m.

The substructure includes 2 abutments and 62 offshore piers with a foundation level between -11 m and -29 m. With regard to the design and construction, differences between the shallow and deep water piers are of a dimensional nature only. The caisson consists of a base plate, from min. 17.5 m x 29.5 m to max. 22.5 m x 34.0 m in size, with the bottom part of the caisson being 6-9 m high and the shaft to level -3.5 m. The up to 7300 tons heavy caissons are placed on compacted, levelled stone beds of 1.5 m - 4 m thickness. After the caissons are sand filled, the pier shafts are placed on top. A typical substructure, including scour protection, is shown in Fig. 3.

Organization

The tender design and documents were formulated for The Great Belt A.S. from December 1987 to April 1988 by a joint venture, CCL, between COWIconsult, Carl Bro Group and Leonhardt, Andrä und Partner. Based on tenders received, an alternative solution, submitted by the European Storebælt Group, ESG, was chosen. ESG is a consortium comprising Højgaard & Schultz A/S, Ballast Nedam Civil Engineering, Taylor Woodrow Construction Ltd., Losinger Ltd., C.G. Jensen A/S and Per Aarsleff A/S. The contract was awarded in June 1989.

The detailed design of the West Bridge structures is performed by the contractor, ESG, parallel to the design activities entrusted to CCL and other consultants. As project manager, CCL coordinates all design activities. The Design Basis is prepared by CCL, including the assessment of soil design parameters and environmental loading.

Great Belt A.S. takes care of the site supervision with assistance from the CCL joint venture regarding the technical aspects.

GEOLOGY

Several site investigations have been carried out since the early 1960s for various Great Belt crossing solutions, including a large number of boreholes and vibrocores, supplemented with extensive seismic surveys. Optimization of the entire link in the late eighties resulted in a relocated alignment of the West Bridge, and thus early investigations were largely located to the north of the present alignment. After the overall geometry of the bridge was known, detailed site investigations were carried out.

The occurrence of the principal subsoil units is shown in Fig. 4.

Danien limestone constitutes the base to any depth important for the foundation. Selandien (Upper Palaeocene) deposits in the form of calcarenite, calcisiltite and marl are encountered at several locations on top of the Danien deposits. Biostratigraphic investigation and other observations suggest that tectonic block-faulted movements took place in a period after the formation of the marl. Most of the upthrow blocks have been removed by erosion, but some can be recognized in the profile where the Selandien sediments are missing, see Fig. 4.

The glacial deposits have been subdivided into three units: Knudshoved Till, Lower TIll and Upper Till. The Knudshoved Till is found exclusively at the western part of the bridge. It is an extremely hard clay and sand till unit. It is characterized by its content of marl and a relatively high plasticity. The plasticity index is 10-20%, and the natural water content is generally high, 12-30%. The $CaCO_3$ content is typically greater than 30%. The Lower Till consists of alternating layers of clay till, sand till and melt water sand. The Upper Till is predominantly a clay till. The Upper and Lower clay tills are essentially low plasticity clays with plasticity indices in the order of 4-8% and liquid limits around 16%. The $CaCO_3$ content in the Upper Till is generally less than 25%, and in the Lower Till generally above 30%. Natural water contents vary generally between 9% and 15%.

With depth, the undrained shear strength of the Upper Till unit has a decreasing tendency in the profile. The strength can vary considerably vertically, typically from 200-250 kN/m^2 at the

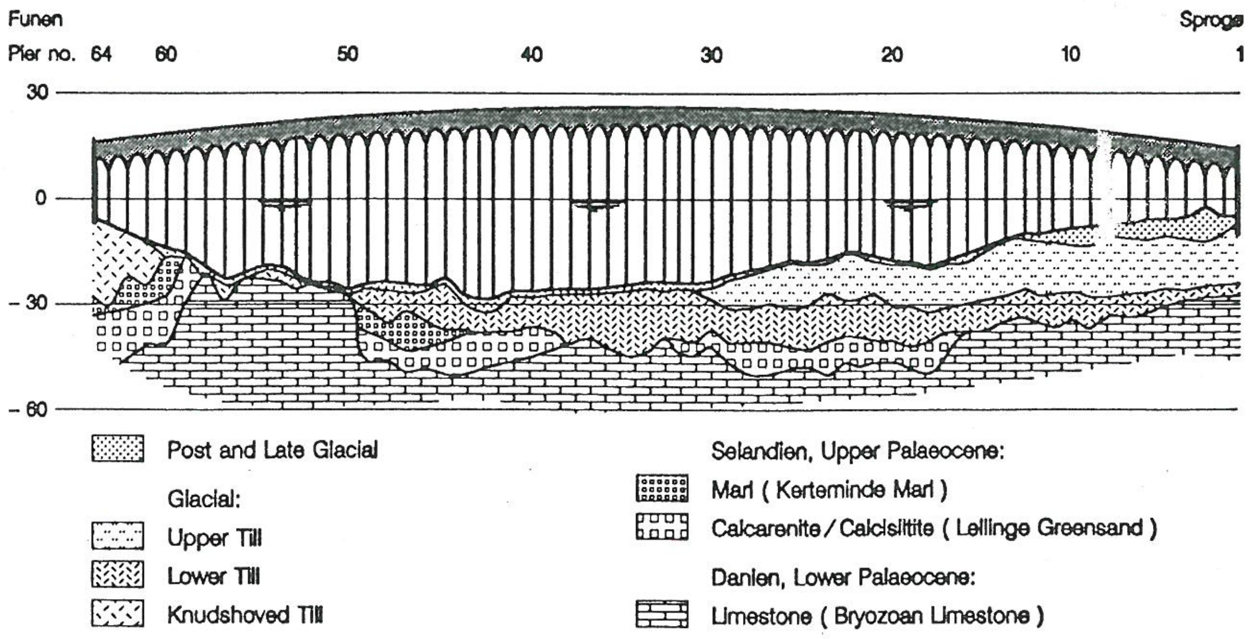

Fig. 4. Longitudinal profile.

upper part and down to values as low as 100-150 kN/m² at the lower part. Large local horizontal variations are also seen.

The Lower Till unit is generally very dense/hard with an undrained shear strength of the clay till typically better than 200-300 kN/m². In some areas the base of the unit is of a lower strength, normally associated with a high content of $CaCO_3$.

According to thermoluminescence dating of sand layers, the Lower Till descends from the next youngest ice age, Saale, or older glaciations. The Upper Till is evaluated to be part of a young terminal moraine complex, of the last ice cover of the area, Weichsel.

The glacial deposits are covered with Late and Post Glacial sediments which are less than 2 m thick and dominated by sandy marine sediments in the main part of the alignment. Thicker deposits including freshwater gyttja are found in the eastern area. The Late and Post Glacial deposits are of little significance for the project. Sediments above level -10 m have been excavated to provide access for marine equipment.

All geotechnical and topographical data are filed in a 3-D computerized information system developed for the Great Belt Project (Porsvig et al., 1989).

DETAILED SITE INVESTIGATIONS

Generally the detailed site investigations comprise 2 geotechnical boreholes and 8 CPTs at each pier. The investigations were performed by Geodan A/S and Fugro-McClelland. A typical site plan is shown in Fig. 5.

For some piers at the eastern part of the bridge an increased number of CPTs, and extra boreholes at a few piers, were deemed appropriate due to large varia-

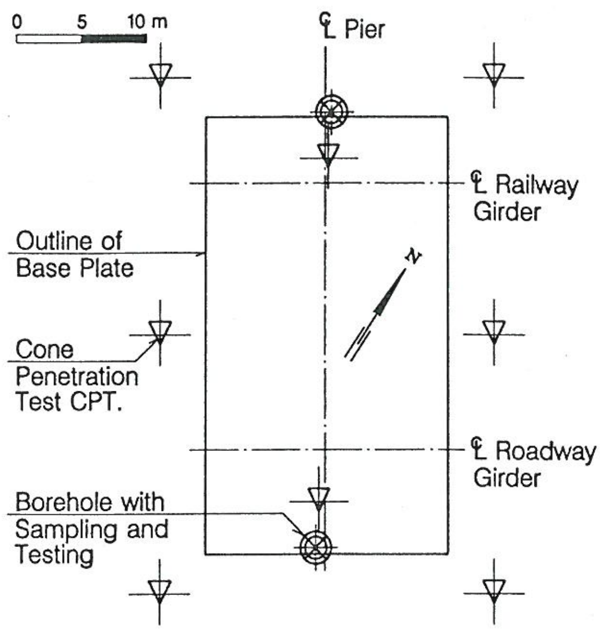

Fig. 5. Locaton of boreholes and CPTs

ation locally in Upper and Lower Till. The additional investigations were made by Fugro-McClelland and Danish Geotechnical Institute.

The boreholes were sunk to 20-40 metres below the seabed. Vane Tests were performed in cohesive material, and Standard Penetration Tests were performed in sand/gravel. Disturbed and intact samples (thin walled tubes) were taken, where advancing was performed by bailing. Cores were taken where coring was possible/necessary. A few Menard Pressuremeter Tests were performed, especially to assess the property of dubious limestone at the limestone ridge, piers 50-58.

Generally CPTs reached refusal at limited (few metres) depth at the western and central part of the bridge. At the eastern part a depth of 20 metres or more was often reached, corresponding to full penetration through the Upper Till.

A ratio has been established between the vane shear strength, c_v, and the CPT cone resistance, q_c, for clay till of the Upper Till unit (Mortensen et al. 1991). For in-situ data (uncorrected), the relation used was:

$$c_v = 0.10\, q_c \qquad (1)$$

LABORATORY INVESTIGATIONS

The laboratory investigations were performed by Danish Geotechnical Institute, Geodan A/S, Norwegian Geotechnical Institute and Aalborg University.

The main aim of the laboratory investigations was to assess the strength and deformation properties of the weak parts of limestone and clay till of the Upper Till unit.

Assessment of the preconsolidation stress σ'_{pc} based on traditional oedometer tests proved to be hardly possible due to the low plasticity and heterogeneous nature of the soils.

Therefore, a large part of the oedometer tests were verified by assessment of preconsolidation stress from triaxial tests.

In the triaxial cell the specimen was preconsolidated to the σ'_{pc} evaluated from oedometer tests. Unloading to in-situ stress was then performed and the triaxial compression test was run. Based on the critical state method, the preconsolidation stress was reassessed. If the latter was higher than the one given to the sample in the laboratory it was assumed that the strength obtained reflected a lower bound value of the in-situ strength. (Jacobsen, 1992).

It has been confirmed that the relation between the vane shear strength, c_v, and the undrained shear strength, c_u, for the clay till is:

$$c_u = c_v \qquad (2)$$

Multistage tests in the laboratory further confirmed the SHANSEP function (Mayne, 1988) between the undrained shear strength, c_u, the in-situ vertical stress, σ'_o, and the preconsolidation stress, σ'_{pc}:

$$c_u = 0.4\, \sigma_o'(\sigma'_{pc}/\sigma'_o)^{0.85} \qquad (3)$$

Based on equations 1, 2 and 3 the undrained shear strength and the preconsolidation stress in the field can be assessed. (Foged and Steenfelt, 1992; Steenfelt and Foged, 1992).

For the Danien lime (stone) it has been demonstrated that soil with a cone resistance, q_c, not less than 1 MPa, increasing with depth, would behave satisfactorily.

The oedometer tests showed that for preconsolidated soils the initial tangential modulus of compressibility, K_t, can be written as:

$$K_t = K_{t,o} + \Delta K_t\, \sigma'_{red} \qquad (4)$$

where $K_{t,o}$ and ΔK_t are constants and σ'_{red} is the minimum vertical stress corresponding to unloading.

SHIP IMPACT AND ICE LOADS

A small amount of triaxial tests were performed with a very high strain rate to failure. The strain rate was also increased for a short while in some of the normal triaxial tests in order to assess the effect of strain rate increase similar to the rate expected at a ship impact.

The results confirm a significant increase in the undrained shear strength as a result of increased rate of strain, see Fig. 6.

Large scale sliding tests have been performed (Bjerregaard Hansen et al. 1991). The sliding resistance in the transition

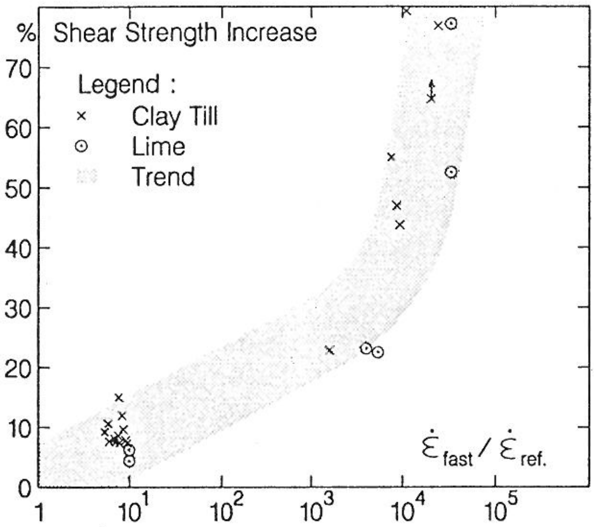

Fig. 6. Shear strength increase as a function of relative strain rate increase.

zone between the stone bed and subsoil of clay till was determined for different degrees of disturbance of the clay till. The tests showed that the ratio between the horizontal sliding resistance, τ_u, and the effective vertical consolidation stress, σ'_c, can be assessed to be (rate 25 mm/sec):

Intact clay till	$\tau_u/\sigma'_c \sim 0.53$
Disturbed clay till	$\tau_u/\sigma'_c \sim 0.46$
Remoulded clay till	$\tau_u/\sigma'_c \sim 0.42$

A comprehensive test programme was fo mulated in order to assess the soil parameters to be used in combination with ice loads, as the nature of ice loading is cyclic and to some extent dynamic.

Due to the dynamic interaction between ice force, structure and soil it was necessary in the first place to assume soil properties which could be used to assess the forces on the structure and the response of the structure before a test programme could be determined. (Christensen et al., 1991; Kristensen et al., 1992).

FOUNDATION DESIGN

The footings of the bridge are designed in limit states with reference to high foundation and safety classes according to "DIF's Code of Practice for Foundation Engineering, DS 415". For layered soil, sliding and torsional moment, additional design rules were developed and used. The following loadings were considered:

- Dead loads
- Train, traffic and other live loads
- Wind loads (10^{-2} event)
- Wave and current action (10^{-2} event)
- Ship impact loads from a 2000 DWT ship
- Ice loads (10^{-4} event)

The ground conditions facilitate direct foundation on the strata below Late and Post Glacial deposits. Hence, substructures are generally designed for foundation on stonebed layers at or slightly below the seabed level. For some piers, however, it was necessary to excavate soft till of the weak Upper Till unit, and to place the foundations on competent Lower Till.

SETTLEMENT MODELS

Two settlement models have been used.

At the western part, where the overconsolidation ratio is high, a linear elastic soil model has been assumed to be sufficient. The Skempton-Bjerrum method (Skempton and Bjerrum, 1957), where the A-factor has been taken as 0,25, has been used for the overall settlement model. Thus, the relation between initial settlements and consolidation settlements can be assessed.

At the eastern part, where the overconsolidation ratio is low, Bjerrum's theory of Delayed Compression (Bjerrum, 1973) has been used for the settlement model. As the theory itself is cumbersome, the initial and consolidation settlements have been assessed using a modulus of compressibility - or compression index - only, and the development of secondary settlements depends on the apparent age of the soil in question.

The settlement model established for weak soil has been substantiated by field model tests. The tests were carried out on Upper Till of low preconsolidation at the island of Sprogø. The main tests were performed on dia. 1.0 m and 2.0 m foundations. The tests did show creep behaviour, basically in accordance with the theory of Bjerrum on Delayed Compression. A time curve is shown in Fig. 7.

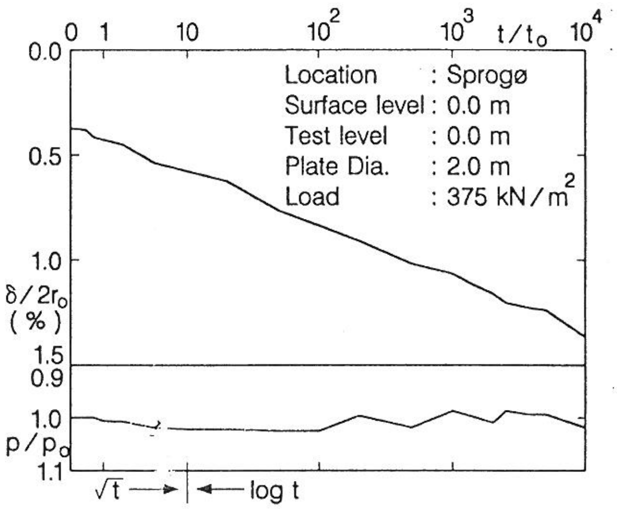

Fig. 7. Time - settlement curve.

SETTLEMENTS OBSERVED

The bridge is constructed from the west. At the time of writing this paper both girders have been placed at Piers Nos. 63-54, and placing of stonebeds and caissons is in progress at Piers Nos. 53 through 45.

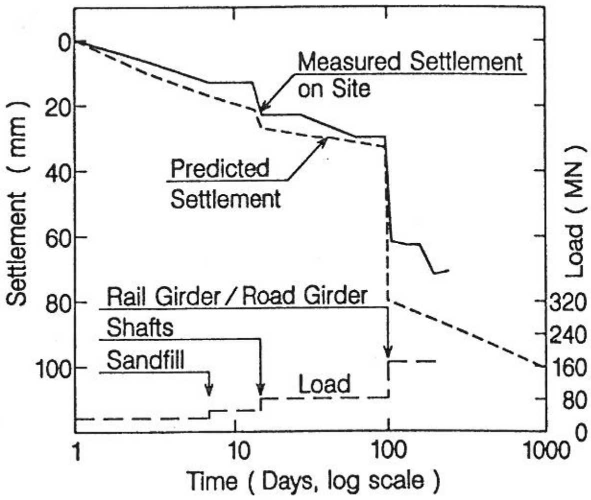

Fig. 8. Predicted and observed settlements of pier No. 60

The erection method with the full dead load instantly applied when the main elements are placed, offers a unique possibility to monitor settlements, especially the relation between initial and consolidation settlements, which is not possible to detect for prolonged construction on glacial tills.

The results of settlement monitoring at Piers 63-54 show a good correspondence between predicted and actual settlements, both being in the order of 4-8 cm from when the caisson is placed until both girders are resting on the pier, see Fig. 8 as an example.

The tilt transverse to the bridge axis is experienced to be 0.5 to 1.5 per mille due to the eccentric load of the road girder. After placing the rail girder the tilt is reduced to less than 0.5 per mille. The tilt parallel to the bridge axis is generally experienced to be less than 0.5 per mille.

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